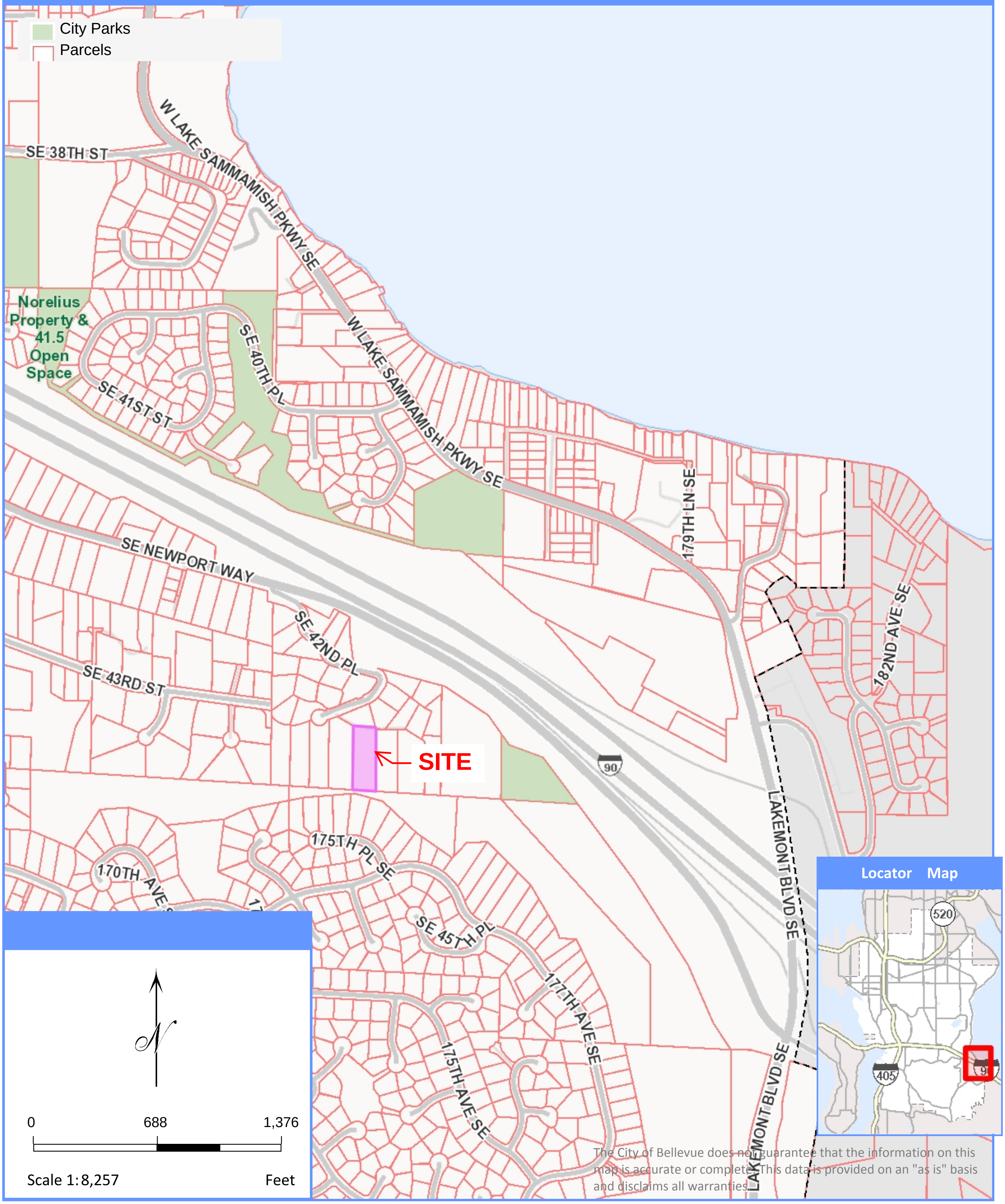
Vicinity Map



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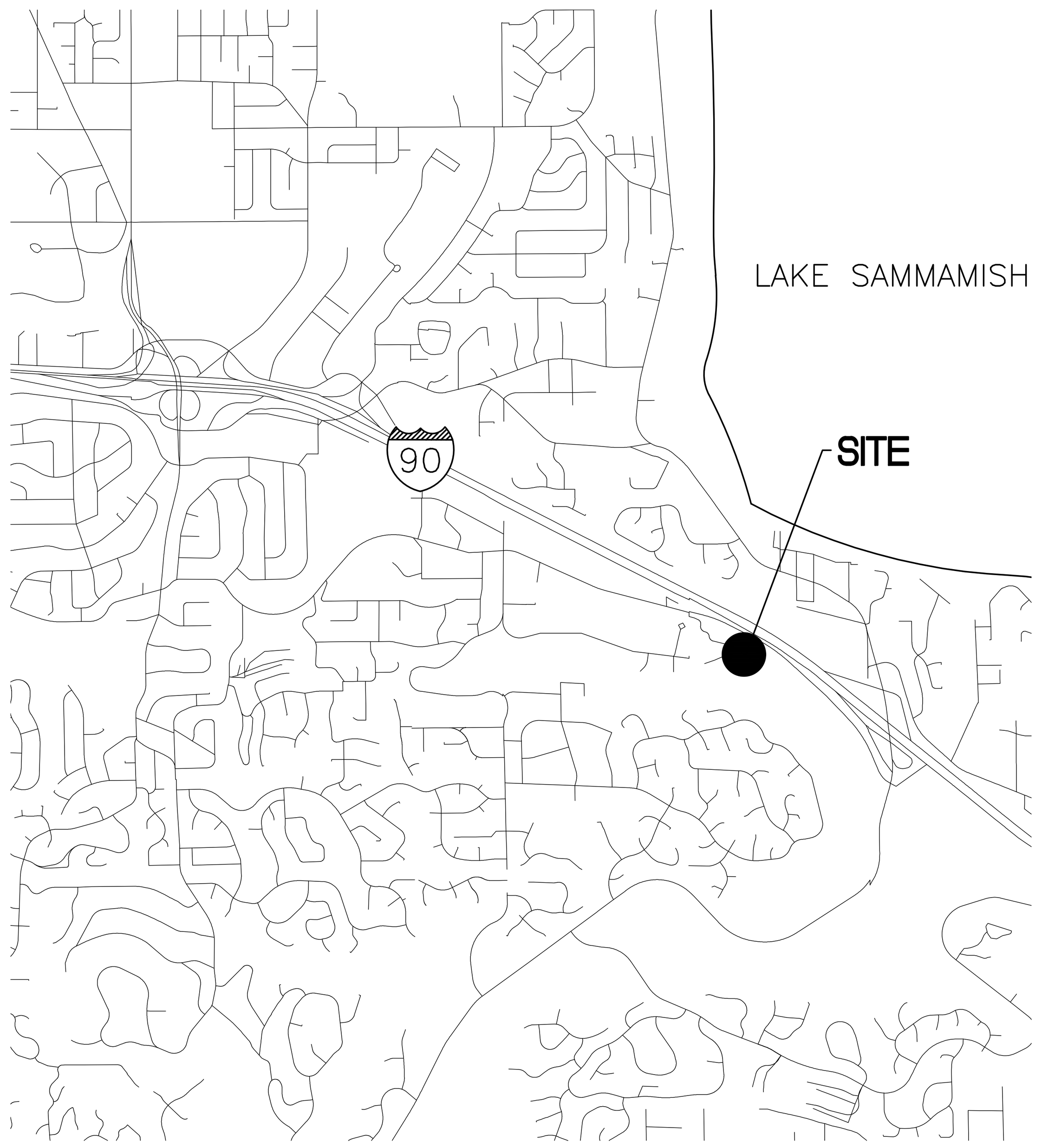
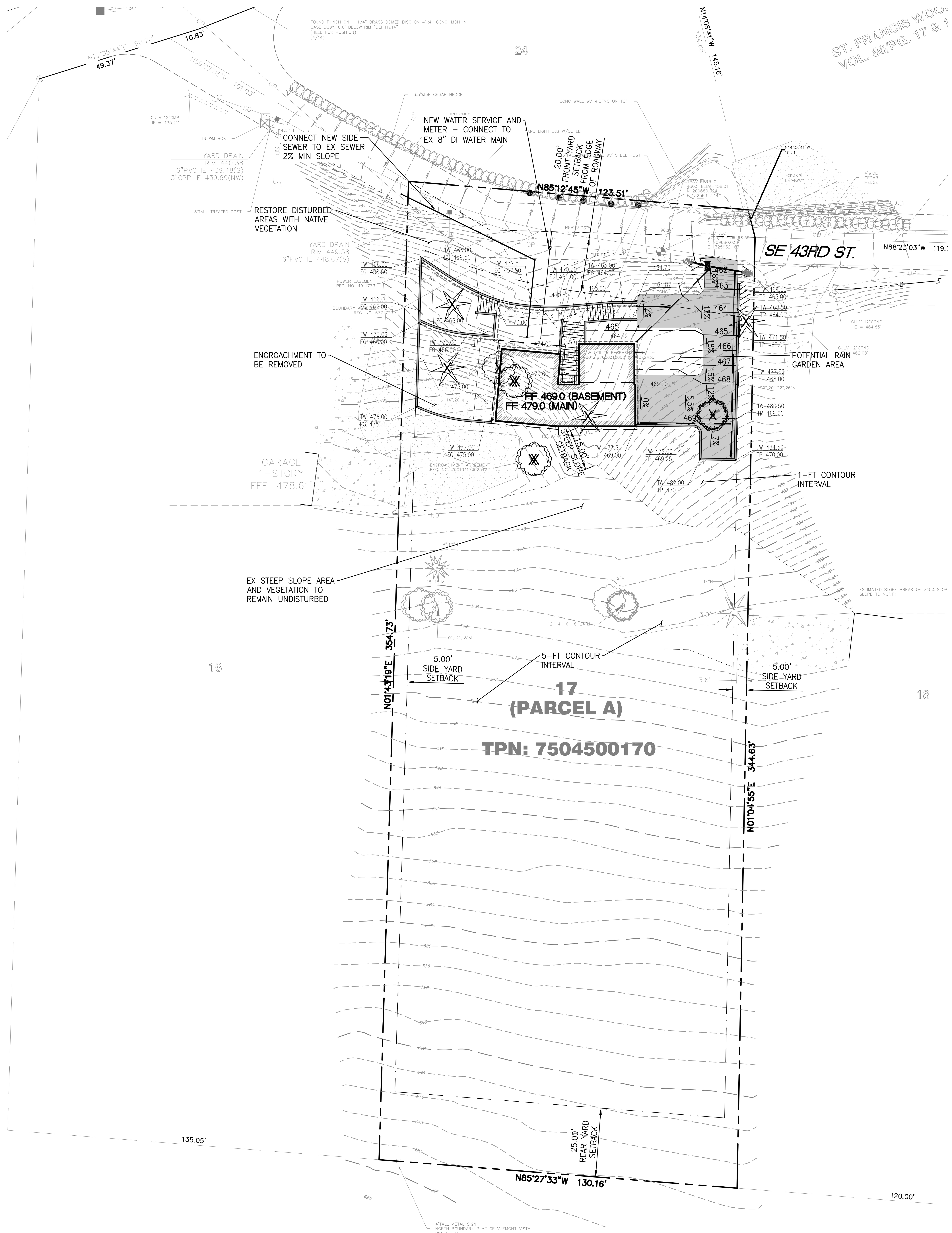


VICINITY MAP
NOT TO SCALE



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designers and builders

LPD engineering pllc
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Suite 201
Seattle, WA 98101
p. 206.725.1211
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LEGEND

- PROPERTY LINE
- EX CONTOUR (INDEX)
- EX CONTOUR
- PROPOSED CONTOUR (INDEX)
- PROPOSED CONTOUR
- SPOT ELEVATION
- TOP OF WALL
- FINISHED GRADE/EXISTING GRADE
- TOP OF PAVEMENT
- FF 78.0 FINISHED FLOOR ELEVATION
- EX TREE TO REMAIN
- EX TREE TO BE REMOVED
- PROPOSED BUILDING
- CONCRETE PAVEMENT
- ASPHALT (AC) PAVEMENT
- SITE WALL
- CATCH BASIN TYPE 1
- STORM DRAINAGE PIPE
- FOOTING/SUBSURFACE DRAIN
- CLEANOUT
- DOWNSPOUTS
- SIDE SEWER PIPE
- SEWER CLEANOUT
- SIDE SEWER CONNECTION
- WATER FITTINGS
- WATER SERVICE LINES
- WATER METER
- WATER SERVICE LINES

PREPARED BY:
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3220 FIRST AVE S, SUITE 300
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Alex.Pfeiffer@battandlear.com

LOT COVERAGE:
LOT AREA 45,365 SF
IMPERVIOUS AREA 3,550 SF

EARTHWORK:
CUT: 600 CY (APPROX)
FILL: 380 CY (APPROX)

PROPERTY OWNER:
DAMIEN + OLIA CATALA
1780 LARCH AVE NE #102
ISSAQUAH, WA 98029

LEGAL DESCRIPTION OF PROPERTY:
ST FRANCIS WOOD
PLAT LOT: 17

PARCEL: 750450-0170
SITE ADDRESS: 17315 SE 42ND CT 98006
QUARTER-SECTION-TOWNSHIP-RANGE: NE-13-24-5

EASEMENTS:
POWER EASEMENT – REC. NO. 4911773

BOUNDRY AGREEMENT LINE – REC. NO. 6371723

ENCROACHMENT AGREEMENT – REC. NO. 20010417002542

20' ROAD AND UTILITY EASEMENT –REC. NO. 7608310801, NO. 7608310802 NO. 6372430

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CATALA HOUSE
17315 SE 42ND COURT BELLEVUE, WA 98006

3220 FIRST AVENUE SOUTH SUITE 300 SEATTLE WA 98134 TELE: 206.301.1999 FAX: 206.328.5428 EMAIL: design@battandlear.com

COB NO:

SITE PLAN B

MARCH 20, 2017

GEOTECHNICAL REPORT
PROPOSED RESIDENCE
17315 Southeast 42nd Court
Bellevue, Washington

PROJECT NO. 14-133.200
March 2017

Prepared for:

Damien and Olia Catala

PanGEO
INCORPORATED

Geotechnical & Earthquake
Engineering Consultants

PanGEO INCORPORATED
Geotechnical & Earthquake Engineering Consultants

March 14, 2017
PanGEO Project No. 14-133.200

Damien and Olia Catala
5551 Lakemont Boulevard Southeast, #1107
Bellevue, Washington 98006

Subject: **Geotechnical Report**
Proposed Residence
17315 Southeast 42nd Court, Bellevue, Washington

Dear Mr. Catala and Mrs. Catala:

As requested, PanGEO, Inc. is pleased to present this geotechnical report to assist the project team with the proposed residence at 17315 Southeast 42nd Court in Bellevue, Washington. In preparing this report, we observed and logged the excavation of five test pits at the site and conducted our engineering analyses. In summary, the site is underlain by localized areas of fill and slopewash overlying weathered siltstone bedrock in the south portion of the site and Vashon till in the north portion of the site.

Portions of the site meet the City of Bellevue criteria for a steep slope hazard area. In our opinion, the standard City of Bellevue 75 foot setback from the toe of steep slope hazard areas can be reduced to 15 feet, provided a debris flow catchment is incorporated into the design to mitigate the risks associated with potential shallow slope failures that may initiate on the steep slope.

In our opinion, the site may be developed generally as planned, with the proposed residence located in the north, relatively level portion of the site. The residence may be supported on a spread footings bearing on competent native soils or on structural fill. Localized footing over-excavation likely will be needed to expose the competent soils for footing construction.

3213 Eastlake Avenue East, Suite B
Seattle, WA 98102
Tel: (206) 262-0370
Fax: (206) 262-0374

We appreciate the opportunity to be of service. Should you have any questions, please do not hesitate to call.

Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer

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ATTACHMENTS:

Figure 1	Vicinity Map
Figure 2	Site and Exploration Plan
Figure 3	Design Lateral Pressures, Cantilevered Soldier Pile Wall
Appendix A	Boring Logs
	Figure A-1 Terms and Symbols for Boring and Test Pit Logs
	Figures A-2 through A-6 Test Pit Logs

GEOTECHNICAL REPORT
PROPOSED RESIDENCE
17315 SOUTHEAST 42ND COURT
BELLEVUE, WASHINGTON

1.0 GENERAL

As requested, PanGEO, Inc. is pleased to present this geotechnical report to assist the project team with the proposed residence at 17315 Southeast 42nd Court in Bellevue, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated September 1, 2016. Our scope of services included reviewing readily available geology map, excavating five test pits, conducting a site reconnaissance, performing our engineering evaluation, and preparing this report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 17315 Southeast 42nd Court in Bellevue, Washington, approximately as shown on Figure 1, Vicinity Map. The site consists of a rectangular-shaped property comprising a total area of 46,173 square feet. The layout of the site is shown on the attached Figure 2.

The site is bordered to the west by a single family residence, to the north by Southeast 42nd Court, to the east by a driveway accessing the residence at 17327 Southeast 42nd Court, and to the south by undeveloped land.

The site is located on the lower portion of the north slope of Cougar Mountain. As such, the site and surrounding area slopes down from north to south with about 180 feet of elevation change across the 355-foot length of the site. The overall slope gradient is about 50 percent. However, the site slopes are not uniform. The slopes in the south two-thirds of the site range from 40 to 80 percent while the slopes in northern third of the site range from 20 to 40 percent.

The site is densely vegetated with big leaf maple and Douglas fir trees and a dense understory of blackberry brambles and sword fern. Plates 1 and 2 on the next page illustrate the general site conditions.

At the time of our field exploration, we noted there was surface water flowing through the central portion of the site. The water is being directed through a shallow swale adjacent to Southeast 42nd Court. We estimated flow rates through the swale at 5 to 10 gallons per minute at the time of our field exploration.

Plate 1: View from northwest corner of the site to the southeast.

Plate 2: View from northeast portion of the site to the southwest.

We understand it is planned to construct a new residence at the site. The proposed residence will be located in the north, more gently sloping portion of the site (see Figure 2). The residence will be three stories in height with the lower level benched into the sloping grade and daylighting to the north. The residence will be accessed from a driveway on the west side of the site. In order to achieve the lower level construction subgrade elevation for the house and driveway, an excavation that is eight to ten feet deep along the south side of the residence that daylights to the north is planned.

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed. In any case PanGEO should be retained to provide a review of the final design to confirm that our geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

3.0 SUBSURFACE EXPLORATIONS

3.1 SITE GEOLOGY

To gain an understanding of the geologic setting at the site, we reviewed the *Geologic Map of the East Half of the Bellevue South 7.5' x 15' Quadrangle, Issaquah Area, King County, Washington* (Booth, 2012). Based on our review, the two primary geologic units in the vicinity of the site are Vashon Till (Geologic Map Unit Qvt) and a Tertiary-age bedrock unit known as the Blakely Formation (Tb).

Vashon till underlies the north half of the site and is comprised of an unsorted mixture of clay, silt, sand and gravel deposited directly by a glacier. Vashon till has been glacially overridden and is typically dense to very dense.

The Blakely Formation consists of a sedimentary deposits comprised of sandstone, conglomerate, airfall tuff, tuffaceous sandstone, and minor siltstone deposited in a marine environment.

3.2 SUBSURFACE EXPLORATION

We excavated five test pits at the site on October 18, 2016. The test pits were excavated using a Bobcat 355 mini-excavator owned and operated by NW Excavating, Inc. under subcontract to PanGEO and were logged by an engineering geologist from PanGEO. The test pits were excavated to a maximum depth of about nine feet below existing grade. The approximate test pit locations were located in the field by measuring from the site boundaries and are shown on Figure 2, Site and Exploration Plan.

Summary test pit logs, included in Appendix A, provide detailed descriptions of the materials encountered, depths to soil contacts, and depths of seepage or caving, if present. The relative in-situ density of cohesionless soils, or the relative consistency of fine-grained soils, was estimated from the excavating action of the excavator, and the stability of the test pit sidewalls. Where soil contacts were gradual or undulating, the average depth of the contact was recorded on the log.

The soils were logged in general accordance with ASTM D-2487 S*tandard Practice for Classification of Soils for Engineering Purposes* and the system summarized on Figure A-1, Terms and Symbols for Boring and Test Pit Logs.

3.3 SOIL CONDITIONS

For a detailed description of the subsurface conditions encountered at each exploration location, please refer to our boring logs provided in Appendix A. The stratigraphic contacts indicated on the boring logs represent the approximate depth to boundaries between soil units. Actual transitions between soil units may be more gradual or occur at different elevations. The descriptions of groundwater conditions and depths are likewise approximate. The following is a generalized description of the soils encountered in the borings.

Topsoil: We encountered a surficial topsoil layer at all of our exploration locations, except Test Pit TP-5. The topsoil ranged from six to twelve inches thick and was comprised of very loose silty sand and sandy silt with organics. The topsoil was characterized by its dark brown color, loose consistency, and the presence of abundant roots and organic debris. This soil layer is not considered suitable for support of foundations, slab-on-grade floors, or pavements. In addition, it is not suitable for use as structural fill, nor should it be mixed with materials to be used as structural fill.

A surficial layer of topsoil was not encountered at Test Pit TP-5, however, we did encounter a 12 inch thick buried topsoil horizon below an approximately 1½ foot thick layer of fill.

Fill: At the locations of test pits TP-1 and TP-5, we encountered fill. These test pits were located in the northwest portion of the site where a prism of fill appears to have been placed to provide a level parking area for the residence at 17301 Southeast 42nd Court.

The fill ranged from 1½ feet thick at Test Pit TP-5 to 6½ feet thick at Test Pit TP-1. The fill consisted of silty sand with gravel and was characterized by its loose condition and the presence of organics, primarily topsoil.

Blakely Formation: Below the fill in Test Pit TP-1, and below the topsoil in Test Pit TP-2, we encountered weathered bedrock comprised of sandstone that is consistent with the description of the Blakely Formation. The weathered bedrock consisted of fine to medium angular gravel- and cobble-sized siltstone in a sandy silt soil matrix.

The bedrock weathering decreased with depth. At the location of Test Pit TP-2 we encountered refusal of our excavation equipment at 4½ feet below grade in hard relatively unweathered sandstone.

Vashon Till (Qvt): Below the topsoil in Test Pit TP-3 and TP-4 and below the fill/buried topsoil horizon in Test Pit TP-5, we encountered silty fine to medium sand with gravel. The soil was typically medium dense, grading to dense at about four feet below grade.

We classified this soil as Vashon till. Test Pits TP-3, TP-4 and TP-5 were all terminated in till.

Our subsurface descriptions are based on the conditions encountered at the time of our exploration. Soil and rock conditions between our exploration locations may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, PanGEO should be requested to reevaluate the recommendations in this report and to modify or verify them in writing prior to proceeding with earthwork and construction.

3.4 Groundwater

Groundwater seepage was encountered in Test Pit TP-1 at seven feet below grade. No seepage was encountered at the other test pit locations. With the planned daylight basement excavation, there is a potential groundwater seepage may be encountered in the site excavations.

Besides groundwater, as discussed above, we observed significant flow of surface water at the time of site visit. Proper control of surface water will need to be incorporated into the overall stormwater management plan for the project.

The designer and contractor should also be aware that groundwater levels will fluctuate depending on the season and precipitation. Typically groundwater levels and seepage rates are greater during the wet season (October through May).

4.0 ENVIRONMENTALLY CRITICAL AREAS CONSIDERATIONS

As part of our study, we conducted a review of potential geologic hazards at the subject site as defined in the City of Bellevue Land Use Code (LUC) Chapter 20, Section 25H.120. Chapter 20 of the LUC identifies two different types of Geologic Hazards that appear applicable to the subject site: (1) Landslide Hazards, and (2) Steep Slope Hazards.

The City's criteria for landslide and steep slope hazard areas and our assessment of the hazard areas with respect to the subject site are provided in the following sections of this report.

4.1 Landslide Hazard Areas

The BCC land use code defines landslide hazard areas as areas of slope of 15 percent or more with more than 10 feet of rise which also have the following characteristics:

- *Areas of historic failures, including those areas designated as quaternary slumps, earthflows, mudflows, or landslides.*

- *Areas that have shown movement during the Holocene Epoch (past 13,500 years) or that are underlain by landslide deposits.*

- *Slopes that are parallel or subparallel to planes of weakness in subsurface materials.*

- *Slopes exhibiting geomorphological features indicative of past failures, such as hummocky ground and back-rotated benches on slopes.*

- *Areas with seeps indicating a shallow ground water table on or adjacent to the slope face.*

- *Areas of potential instability because of rapid stream incision, stream bank erosion, and undercutting by wave action.*

In order to evaluate the landslide hazard at the subject site, we reviewed the *Geologic Map of Seattle and Vicinity, Washington* (Waldron, 1962), *Geologic Map of the East Half of the Bellevue South 7.5' x 15' Quadrangle, Issaquah Area, King County, Washington* (Booth, 2012) and historical slope stability information in our library and files. Based on our review, the site is not mapped as containing Quaternary age slumps, earthflows, mudflows or landslides.

We also conducted a reconnaissance of the site and site slopes on October 18, 2016. The purpose of our reconnaissance was to review the condition of the site slopes and identify indications of historical slope instability, which included:

- Bowl-shaped topography;
- Irregular or hummocky topography;
- Tension cracks, scarps, or other indicators of ground movement;
- Leaning or pistol-butted trees;
- Distressed vegetation;
- Vegetation of markedly different ages or types (i.e., a swath of young alders and blackberries in an otherwise mature forest);
- "Fresh" looking soil deposited at the base of steep slopes;
- Disturbed or destroyed anthropogenic features, such as fence lines that have been displaced;
- Hillside seeps or springs; and
- Ponding water/sag ponds.

Based on the conditions observed during our reconnaissance, we did not observe any indications of historical slope instability. We also did not encounter soils in our test pits that are consistent with landslide deposits.

The native soils underlying the site consist of sandstone bedrock and glacial till, a soil unit that has relatively high strength. We did not encounter indications of planes of weakness or preferential failure surfaces.

During our field exploration, we observed there is surface water flowing through the central portion of the site. We estimate the flow at five to ten gallons per minute. We could not determine whether the source of the surface water was due to recent heavy precipitation or emergent groundwater.

The site is not located adjacent to a watercourse or water body that could result in erosion or undercutting of the slope.

Based on our review the observed site conditions, in our opinion, the subject site does not meet the Bellevue LUC definition of a Landslide Hazard Area.

4.2 STEEP SLOPE HAZARD AREAS AND SETBACK

The City of Bellevue defines Steep Slope Hazard Areas as:

> *"slopes of 40 percent or more that have a rise of at least 10 feet and exceed 1,000 square feet in area"*

The south portion of the site has a slope gradient in excess of 40 percent, a slope height of more than 140 feet and exceeds 1,000 square feet in area. As such, the slope in the south portion of the site meets the criteria of a Bellevue LUC defined steep slope hazard area. The extent of the steep slope hazard area is approximately shown on Figure 2.

Bellevue LUC Section 20.25H.120B Geologic Hazard Area Buffers specifies a 75-foot construction setback from the toe of steep slope areas, but the setback may be reduced if a geotechnical engineering study is performed to evaluate if a reduced setback is appropriate.

Based on the results of our field investigation, the steep slope is underlain by thin surficial layer of loose to medium dense soils overlying sandstone bedrock. The uppermost two to three of surficial soils may be susceptible to shallow surficial slope failures, but should not be susceptible to deep-seated global slope failures.

A shallow slope failure, or debris flow failure, consists of soil and water and will contain vegetation and other debris. This type of slope failure will flow down the slope as a fluid mass, however when the failure reaches a break in slope, the water constituent in the failure will drain out and the mass will slow down and stop.

In order to reduce the construction setback from the south slope, we recommend providing a debris flow catchment on the south side of the proposed residence. The catchment may consist of a landscaped soil berm or a catchment wall incorporated into the structure. A horizontal distance of 10 feet should be maintained between the proposed house and the catchment wall/berm.

If a berm is used, the berm should consist of free draining aggregate such as quarry spalls with a minimum height of five feet. The crest of the berm should be at least five feet wide and the side slopes inclined no steeper than 2H:1V. The berm may be landscaped.

Alternatively, a debris flow catchment may be constructed south of the proposed residence. The wall may consist of a gravity wall (example: gabion wall) or a structural wall (example: soldier pile wall). The debris flow catchment should consist of extending the south retaining wall of the residence to a height of three feet above finished grade. The catchment portion of the wall should be designed for a dynamic force of 200 pounds per square foot plus a rectangular pressure distribution of 60H pcf, where H is the height of the catchment.

Consideration will also need to be given to the placement of door and window openings on the south side of the residence. Window and door openings should not be located within the height of the catchment area, so they do not allow debris to enter the residence.

In our opinion, if a debris flow catchment is provided, either as a landscaped berm or incorporated into the structure, the steep slope building setback may be reduced to 15 feet.

Alternatively, in lieu of a detached catchment wall, the lower 10 feet of the above-ground portion of the south building wall should be designed to withstand the same pressure as a catchment wall.

4.3 Wetland and Habitat Critical Areas

An evaluation of wetland and habitat critical areas were not included as part of our study.

5.0 INFILTRATION CONSIDERATIONS

Based on our subsurface exploration, the soils underlying the site consist of fill, Vashon till, and sandstone of the Blakely formation bedrock.

Based on our experience with this soil and bedrock, these materials would not be suitable as receptor soils for infiltration. Additionally, due to the steep slopes at the site, we would not recommendation infiltrating stormwater. Stormwater should be collected and discharged to the storm drainage system.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Design Parameters

The 2012/2015 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Table 1 below provides seismic design parameters for the site that are in conformance with the 2012/15 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

Table 1 – Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. [g] S_S	Spectral Acceleration at 1.0 sec. [g] S_1	Site Coefficients		Design Spectral Response Parameters		Control Periods [sec.]	
			F_a	F_v	S_{DS}	S_{D1}	T_O	T_S
D	1.341	0.511	1.000	1.500	0.894	0.511	0.572	0.114

Liquefaction Potential: Liquefaction is a process that can occur when soils lose shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration results in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid. Soils with a potential for liquefaction are typically cohesionless, predominately silt and sand sized, must be loose, and be below the groundwater table. The proposed building area is underlain by medium dense to very dense silty sand with gravel soils and moderately weathered to unweathered sandstone without a defined groundwater table. Based on these conditions, in our opinion the liquefaction potential of the site is negligible and design considerations related to soil liquefaction are not necessary for this project.

6.2 Building Foundations

Based on the subsurface conditions encountered at the site and our understanding of the planned development, it is our opinion the proposed residence may be supported on spread footings. The footings should bear on undisturbed native soil underlying the site or on properly compacted structural fill placed on undisturbed native soil. Where loose soils are exposed at the construction subgrade elevation, the loose soils should be overexcavated and replaced with structural fill.

Exterior foundation elements should be placed at a minimum depth of 18 inches below final exterior grade. Interior spread foundations should be placed at a minimum depth of 12 inches below the top of concrete slabs.

We recommend a maximum allowable soil bearing pressure of 3,000 pounds per square foot (psf) be used to size the footings. The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces. Continuous and individual spread footings should have minimum widths of 18 and 24 inches, respectively.

Footings designed and constructed in accordance with the above recommendations should experience total settlement of about one inch and differential settlement of less than ½ inch. Most of the anticipated settlement should occur during construction as dead loads are applied.

6.2.1 Lateral Resistance

Lateral loads on the structures may be resisted by passive earth pressure developed against the embedded portion of the foundation system and by frictional resistance between the bottom of the foundation and the supporting subgrade soils. For footings bearing on the medium dense to very dense silty sand with gravel, a frictional coefficient of 0.30 may be used to evaluate sliding resistance developed between the concrete and the subgrade soil. Passive soil resistance may be calculated using an equivalent fluid weight of 300 pcf, assuming foundations are backfilled with structural fill. The above values include a factor of safety of 1.5. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

6.2.2 Perimeter Footing Drains

Footing drains should be installed around the perimeter of the residence, at or just below the invert of the footings. Under no circumstances should roof downspout drain lines be connected to the footing drain systems. Roof downspouts must be separately tightlined to appropriate discharge locations. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

6.2.3 Footing Subgrade Preparation

All footing subgrades should be in a dense condition prior to setting forms and placing reinforcing steel. Any loose or softened soil should be removed from the footing excavations. The adequacy of the footing subgrade soils should be verified by a representative of PanGEO, prior to placing forms or rebar.

6.3 FLOORS SLABS

The floor slabs for the proposed residence may be constructed using conventional concrete slab-on-grade floor construction. The floor slab should be supported on competent native soil or on structural fill. Any over-excavations, if needed, should be backfilled with structural fill.

Interior concrete slab-on-grade floors should be underlain by a capillary break consisting of at least of 4 inches of pea gravel or compacted ¾-inch, clean crushed rock (less than 3 percent fines). The capillary break material should meet the gradational requirements provided in Table 2, below.

Table 2 – Capillary Break Gradation

Sieve Size	Percent Passing
¾-inch	100
No. 4	0 – 10
No. 100	0 – 5
No. 200	0 – 3

The capillary break should be placed on the subgrade that has been compacted to a dense and unyielding condition.

A 10-mil polyethylene vapor barrier should also be placed directly below the slab. Construction joints should be incorporated into the floor slab to control cracking.

6.3.1 Underslab Drainage

Due to the shallow groundwater encountered in Test Pit TP-1 and presence of surface water, an underslab drainage system is recommended below the lower parking level in the northwest portion of the site. The underslab drainage system should consist of gravel-filled trenches installed with a horizontal spacing of no more than 15 feet. The trenches should be at least one foot deep and one foot wide and lined with a non-woven filter fabric such as Mirafi 140N, or equivalent. A 4-inch perforated PVC (Schedule 35 minimum) pipe should be placed at the bottom of the trenches. The trenches should be backfilled with Gravel Backfill for Drains as defined in Section 9-03.12(4) of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT 2016), or an approved equivalent. The collected water may be tied into the footing drain system for discharge.

6.4 RETAINING WALL DESIGN PARAMETERS

6.4.1 Retaining and Basement Walls

Retaining and basement walls should be designed to resist the lateral earth pressures exerted by the soils behind the wall. Proper drainage provisions should also be provided behind the walls to intercept and remove groundwater that may be present behind the wall.

Cantilever walls should be designed for an equivalent fluid pressure of 45 pcf for a level backfill condition behind the walls assuming the walls are free to rotate. If the walls are restrained at the top from free movement, an equivalent fluid pressure of 55 pcf should be used for a level backfill condition behind the walls. Permanent walls should be designed for an additional uniform lateral pressure of 7H psf for seismic loading, where H corresponds to the height of the buried depth of the wall.

The recommended lateral pressures assume that the backfill behind the walls consists of a free draining and properly compacted fill with adequate drainage provisions.

6.4.2 Surcharge

Surcharge loads, where present, should also be included in the design of retaining walls. A lateral load coefficient of 0.3 should be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half the wall height.

6.4.3 Lateral Resistance

Lateral forces from seismic loading and unbalanced lateral earth pressures may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and by friction acting on the base of the wall foundation. Passive resistance values may be determined using an equivalent fluid weight of 300 pcf. This value includes a factor of safety of 1.5, assuming the footing is backfilled with structural fill. A friction coefficient of 0.30 may be used to determine the frictional resistance at the base of the footings. The coefficient includes a factor of safety of 1.5.

6.4.4 Wall Drainage

Provisions for wall drainage should consist of a 4-inch diameter perforated drainpipe placed behind and at the base of the wall footings, embedded in 12 to 18 inches of clean crushed rock or pea gravel wrapped with a layer of filter fabric. A minimum 18-inch wide zone of free draining granular soils (i.e. pea gravel or washed rock) is recommended to be placed adjacent to the wall for the full height of the wall. Alternatively, a composite drainage material, such as Miradrain 6000, may be used in lieu of the clean crushed rock or pea gravel. The drainpipe at the base of the wall should be graded to direct water to a suitable outlet.

6.4.5 Wall Backfill

Wall backfill should consist of imported, free draining granular material meeting the requirements of Gravel Borrow as defined in Section 9-03.14(1) of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT, 2016). In areas where space is limited between the wall and the face of excavation, pea gravel may be used as backfill without compaction.

In our opinion, the predominately silty sand soils underlying the site are not suitable for use as wall backfill.

Wall backfill should be moisture conditioned to near optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D-1557. Within 5 feet of the wall, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density.

6.5 Surface Water Drainage

At the time of our field exploration, there was surface water flowing through the north portion of the site. It is likely the surface water is related to recent heavy precipitation that occurred around the time of our field exploration.

The control of surface water should be a consideration during the design of subsurface and surface water drainage measures.

6.6 Permanent Cut and Fill Slopes

Based on the anticipated soil that will be exposed in the planned excavation, we recommend permanent cut and fill slopes be constructed no steeper than 2H:1V (Horizontal:Vertical).

7.0 TEMPORARY SOLDIER PILE SHORING

The proposed excavation will extend to a depth of eight to ten feet below grade. Due to the steep slope on the south side of the planned building area, a conventional open cut with temporary slopes may not catch grade without encroaching on the steep slope area. In order to reduce the footprint of the excavation, temporary shoring can be used.

In our opinion, a soldier pile wall with timber lagging is the most appropriate temporary shoring option for these soil conditions. The shoring system should be designed to provide adequate protection for the workers, adjacent structures, utilities, and other facilities. Excavations should be performed in accordance with the current requirements of WISHA. Construction should proceed as rapidly as feasible, to limit the time temporary excavations are open.

A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed excavation wall, spanned by timber lagging. Prior to the start of excavation, the steel beams are installed in holes drilled to a design depth and then backfilled with lean mix or structural concrete. As the excavation proceeds downward and the steel piles are subsequently exposed, timber lagging is installed between the piles to further stabilize the walls of the excavation. In general, tiebacks are typically used for wall heights greater than about 12 feet to achieve a more economical design. If it is planned to use tiebacks, we can provide additional recommendations for tieback design and testing.

7.1 WALL DESIGN PARAMETERS

We recommend the earth pressures depicted on Figure 3 be used for the shoring design.

Vertical Capacity – The vertical capacity of the soldier piles should be determined using an allowable skin friction value of 0.5 ksf for the portion of the pile below the bottom of the excavation, and an allowable end soil bearing capacity value of 10 ksf.

7.2 Lagging

Lagging design recommendations are presented on Figure 3. Lagging located within 10 feet of the top of the shoring which may be subjected to surcharge loads from construction equipment or material storage should be designed for an additional uniform lateral surcharge pressure of 200 psf. This pressure approximately corresponds to a vertical uniform surcharge load of 500 psf at the top of the wall.

Point loads located close to the top of the wall may apply additional loads to the lagging. These loads will need to be individually analyzed.

We recommend that voids behind the lagging be backfilled with CDF.

7.3 Baseline Survey and Monitoring

Ground movements may occur as a result of excavation activities. As such, ground surface elevations of the adjacent properties and city streets should be documented prior to commencing earthwork to provide baseline data. As a minimum, optical survey points should be established at the following locations:

- The top of every other soldier pile. These monitoring points should be monitored twice a week during excavation, once a week after the footings are cast against the shoring wall, and can be stopped after the permanent walls and floor diaphragms are in place. The monitoring frequency may be reduced based on the monitoring results.

The monitoring program should include changes in both the horizontal (x and y directions) and vertical deformations. The monitoring should be performed by the contractor or the project surveyor, and the results be promptly submitted to PanGEO for review. The results of the monitoring will allow the design team to confirm design parameters, and for the contractor to make adjustments if necessary.

We also recommend that the existing conditions along adjacent private properties be photo-documented prior to commencing earthwork at the site.

8.0 EARTHWORK CONSIDERATIONS

8.1 Temporary Excavations

In order to achieve construction subgrade elevations, we anticipate cuts of three to five feet deep may be needed. Temporary excavations should be constructed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. It is our opinion temporary excavations in the fill soils and underlying native soils may be cut at a maximum 1H:1V inclination.

Temporary excavations should be evaluated in the field during construction based on actual observed soil conditions. If seepage is encountered, excavation slope inclinations may need to be reduced. During wet weather, the cut slopes may need to be flattened to reduce potential erosion or should be covered with plastic sheeting.

8.2 Material Reuse

The native soils underlying the site are moisture sensitive, and will become disturbed and soft when exposed to inclement weather conditions. We do not recommend reusing the native soils as structural fill. If it is planned to use the native soil in non-structural areas, the excavated soil should be stockpiled and protected with plastic sheeting to prevent it from becoming saturated by precipitation or runoff.

8.3 Structural Fill and Compaction

Structural fill, if needed, should be free of organic and inorganic debris, be near the optimum moisture content, and capable of being compacted to the recommended requirements described below. The native soils that underlie the site would be suitable for use as structural fill during dry weather. Fill for use during wet weather should consist of a granular fill consisting of well graded material free of organic material, with less than 5 percent fines (that portion of the soil that passes the US No. 200 sieve).

Structural fill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and compacted to at least 95 percent maximum density, determined using ASTM D 1557 (Modified Proctor). The procedure to achieve proper density of a compacted fill depends on the size and type of compacting equipment, the number of passes, thickness of the lifts being compacted, and certain soil properties. If the excavation to be backfilled is constricted and limits the use of heavy equipment, smaller equipment can be used, but the lift thickness will need to be reduced to achieve the required relative compaction.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with high fines contents are particularly susceptible to becoming too wet and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.

8.4 Wet Weather Construction

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. The following procedures are best management practices recommended for use in wet weather construction:

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing the 0.75-inch sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Geotextile silt fences should be installed at strategic locations around the site to control erosion and the movement of soil.
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

8.5 Erosion Considerations

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area leaving the immediate work site. Temporary erosion control may require the use of hay bales on the downhill side of the project to prevent water from leaving the site and potential storm water detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to a positive and permanent discharge system.

Permanent control of surface water should be incorporated in the final grading design. Adequate surface gradients and drainage systems should be incorporated into the design such that surface runoff is collected and directed away from the structure to a suitable outlet. Potential issues associated with erosion may also be reduced by establishing vegetation within disturbed areas immediately following grading operations.

9.0 ADDITIONAL SERVICES

To confirm that our recommendations are properly incorporated into the design and construction of the proposed buildings, PanGEO should be retained to conduct a review of the final project plans and specifications, and to monitor the construction of geotechnical elements. The City of Bellevue, as part of the permitting process, will also require geotechnical construction inspection services. PanGEO can provide you a cost estimate for construction monitoring services at a later date.

10.0 CLOSURE

We have prepared this report for Damien and Olia Catala and the project design team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of services.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our services specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

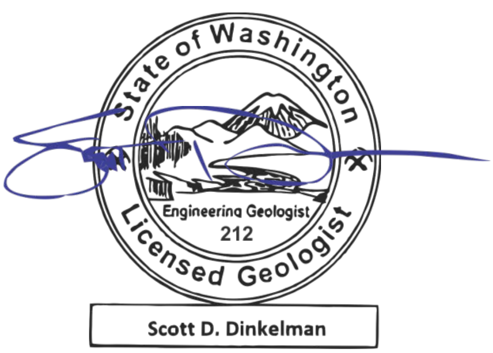
This report has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

Sincerely,

PanGEO, Inc.



Scott D. Dinkelman, LEG, LHG
Senior Engineering Geologist

Siew L Tan, P.E.
Principal Geotechnical Engineer

11.0 REFERENCES

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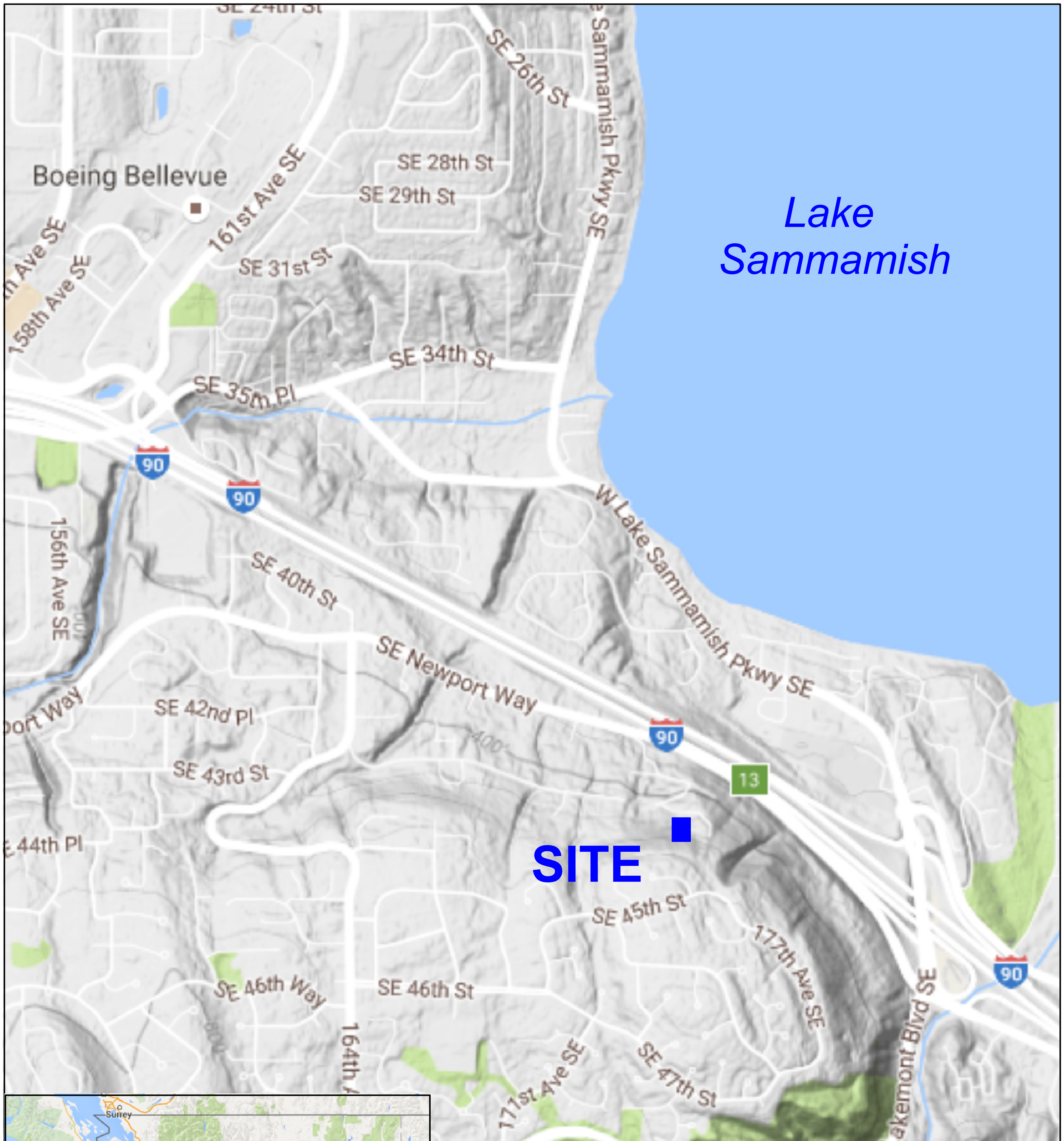
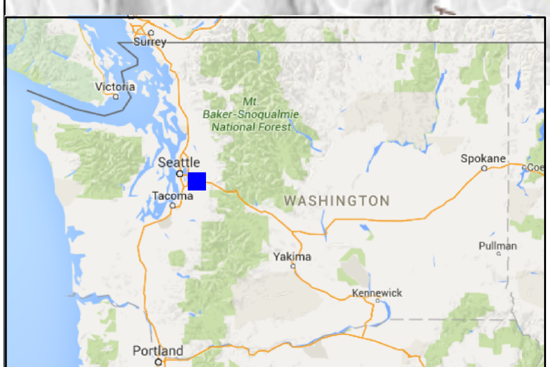




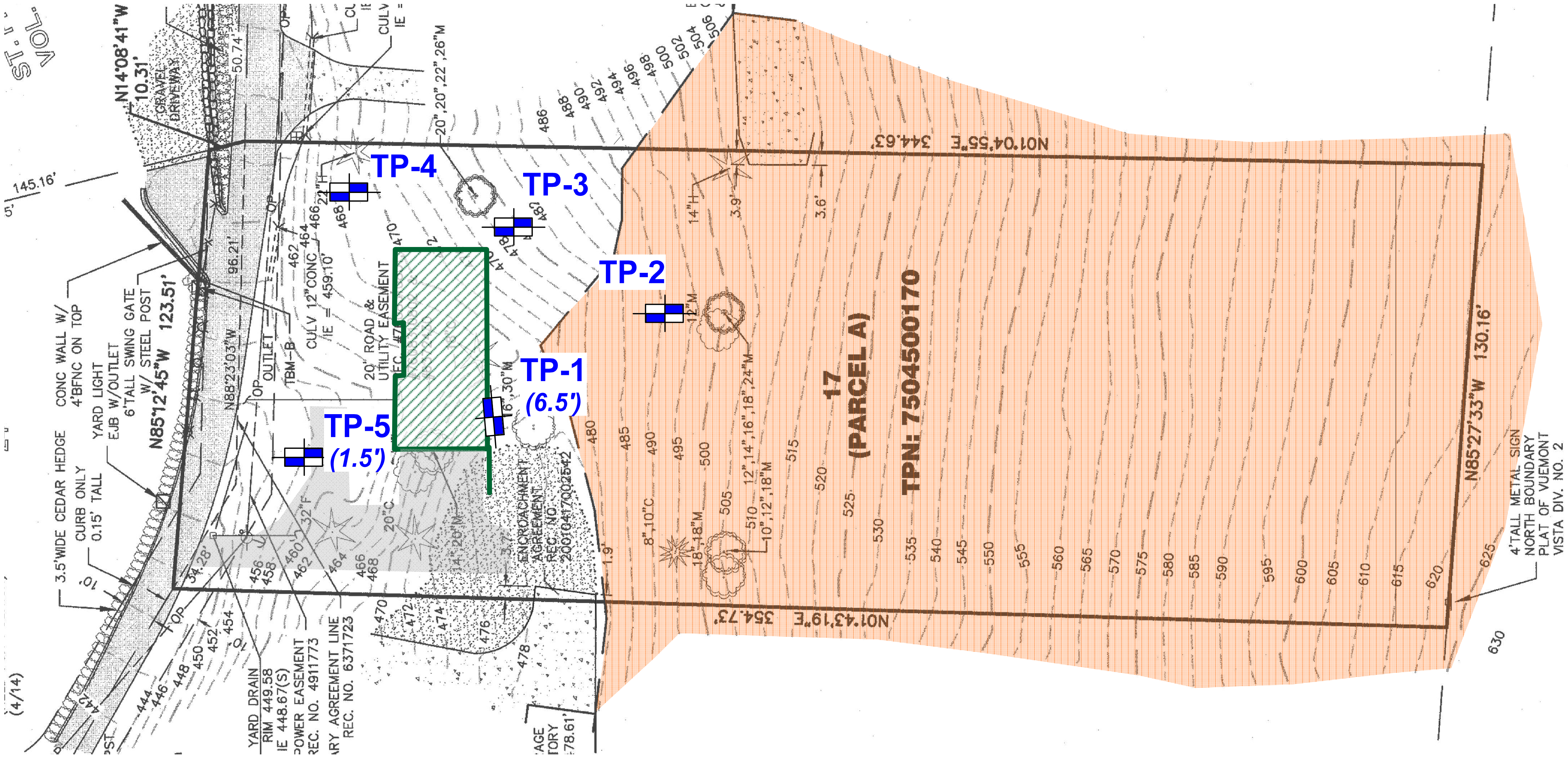
Base Map: Google Maps

NORTH

Not to Scale

PanGEO INCORPORATED	**Proposed Residence** **17315 SE 42nd Court** **Bellevue, WA**	**VICINITY MAP** Project No. **14-133.100** Figure No. **1**

file.grf w/ file.dat 12/5/16 (10:29) SDD



LEGEND:

Subject Site

Proposed Residence

Approximate Extent 40 Percent and Steeper Slopes

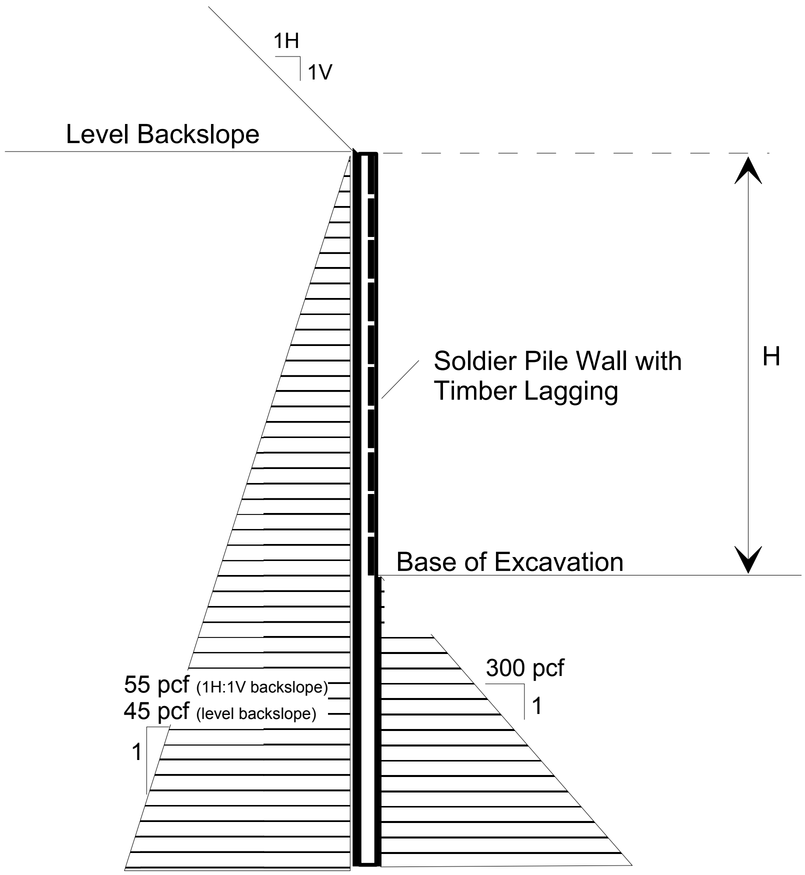
Approximate Test Pit Location, PanGEO, Inc., October 2016
(Fill Thickness in Feet)

Approx. Scale (feet)
0 20 40

NORTH

PanGEO INCORPORATED	Proposed Residence 17315 SE 42nd Ct Bellevue, WA	SITE AND EXPLORATION PLAN Project No. 14-133.200 Figure No. 2

13-011_Fig 2 Site & Exploration Plan.grf 12/5/16 JCR



Notes:
1. Minumum embedment should be at least 10 feet below bottom of excavation.
2. A factor of safety of 1.5 has been applied to the recommended passive pressure values. No factor of safety has been applied to the recommended active earth pressure values.
3. Active pressures should be applied over the full width of the pile spacing above the base of the excavation, and over one pile diameter below the base of the excavation.
4. If surcharges will apply to the walls, PanGEO should be consulted to provide additional design parameters.
5. Passive pressure should be applied to two times the diameter of the soldier piles.
6. Neglect passive pressure in upper two feet to account for disturbance below the wall.
7. Refer to report text for additional discussions.

PanGEO INCORPORATED	**Proposed Residence 17315 SE 42nd Court Bellevue, WA**	**DESIGN LATERAL PRESSURES TEMPORARY CANTILEVERED SOLDIER PILE WALL**
		Project No. 14-133.200 Figure No. 3

EP diagram.grf 12/5/16 (10:35) SDD

APPENDIX A

TEST PIT LOGS

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes:

1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered:	Units of material distinguished by color and/or composition from material units above and below	**Fissured:**	Breaks along defined planes
Laminated:	Layers of soil typically 0.05 to 1mm thick, max. 1 cm	**Slickensided:**	Fracture planes that are polished or glossy
Lens:	Layer of soil that pinches out laterally	**Blocky:**	Angular soil lumps that resist breakdown
Interlayered:	Alternating layers of differing soil material	**Disrupted:**	Soil that is broken and mixed
Pocket:	Erratic, discontinuous deposit of limited extent	**Scattered:**	Less than one per foot
Homogeneous:	Soil with uniform color and composition throughout	**Numerous:**	More than one per foot
		BCN:	Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MONITORING CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Test Pit No. TP-1

Elevation: 474 Feet
Date: October 18, 2016
Surface Vegetation: Maple and Douglas fir trees, sword fern, blackberries

Depth (feet)	USCS	Soil Description
0 - 0.5	Topsoil	TOPSOIL: Loose, dark brown silty SAND with organics, moist
0.5 - 6.5	FILL	FILL: Loose, grayish brown silty fine to medium SAND with fine to coarse subangular gravel, moist to wet • Mottled color • Contains roots up to two inches in diameter • Contains organic debris, topsoil
6.5 – 9.0	BEDROCK	BLAKELY FORMATION (highly weathered): SANDSTONE, fine grained, amorphous, reddish brown and gray, weak, highly weathered, massive • Contains moderately weathered to unweathered, medium strength, cobble sized clasts of sandstone • Perched groundwater seepage at 7, moderate flow • Becomes less weathered, hard at 9 feet below grade

Test Pit TP-1: View of west side of Test Pit TP-1. Completed depth of 9 feet below grade.

Test Pit TP-1: Highly weathered siltstone from about 9 feet below grade.

Test pit terminated at 9.0 feet below grade.
Moderate groundwater seepage encountered at 7 feet during excavation.

Test Pit No. TP-2

Elevation: 500 Feet
Date: October 18, 2016
Surface Vegetation: Maple and Douglas fir trees, sword fern, blackberries

Depth (feet)	USCS	Soil Description
0 – 1.0	Topsoil	TOPSOIL: Loose, dark brown sand SILT with organics, moist
1.0 – 2.5	ML	SLOPEWASH: Loose, light brown SILT, trace sand, with angular fine to coarse siltstone gravel, moist • Contains roots to three inches in diameter
2.5 – 4.5	BEDROCK	BLAKELY FORMATION (weathered): SANDSTONE, fine grained, amorphous, reddish brown and gray, weak, highly weathered, massive • Excavation spoils consist of medium grained angular gravel sized clasts of siltstone • Contains fine roots • Test Pit terminated at 4.5 feet below grade in hard siltstone

Test Pit TP-2: View of excavation spoils consisting of angular sandstone gravel sized particles. Material is from about four feet below grade.

Test pit terminated at 4.5 feet below grade.
No groundwater seepage encountered during excavation.

Test Pit No. TP-3

Elevation: 476 Feet
Date: October 18, 2016
Surface Vegetation: Maple and Douglas fir trees, sword fern, blackberries

Depth (feet)	USCS	Soil Description
0 - 0.5	Topsoil	TOPSOIL: Loose, dark brown silty SAND with organics, moist
0.5 - 6.5	SM	VASHON TILL: Medium dense, grayish brown silty fine to medium SAND with medium to coarse well rounded gravel, moist • Contains roots up to two inches in diameter • Becomes dense at 4 feet below grade • Becomes very dense at 6 feet below grade

Test Pit TP-3: View of spoils pile from Test Pit TP-3.

Test pit terminated at 6.5 feet below grade.
No groundwater seepage encountered during excavation.

Test Pit No. TP-4

Elevation: 468 Feet
Date: October 18, 2016
Surface Vegetation: Maple and Douglas fir trees, sword fern, blackberries

Depth (feet)	USCS	Soil Description
0 - 0.5	Topsoil	TOPSOIL: Loose, dark brown silty SAND with organics, moist
0.5 - 7	SM	VASHON TILL: Medium dense, reddish brown silty fine to medium SAND with medium to coarse well rounded gravel, moist • Contains roots up to two inches in diameter • Becomes gray, dense at 3 feet below grade • Becomes very dense at 6 feet below grade

Test Pit TP-4: View of south end of Test Pit TP-4. Test Pit is approximately 7 feet deep.

Test pit terminated at 7 feet below grade.
No groundwater seepage encountered during excavation.

Test Pit No. TP-5

Elevation: 460 Feet
Date: October 18, 2016
Surface Vegetation: Maple and Douglas fir trees, sword fern, blackberries

Depth (feet)	USCS	Soil Description
0 – 1.5	FILL	FILL: Loose, grayish brown silty fine to medium SAND, some fine to coarse gravel, moist to wet
1.5 - 2.5	TOPSOIL	TOPSOIL: Loose, dark brown silty SAND with organics, moist
2.5 - 6	SM	VASHON TILL: Medium dense, reddish brown silty fine to medium SAND with medium to coarse well rounded gravel, moist • Becomes gray, dense at 4 feet below grade

Test Pit TP-5: View of south end of Test Pit TP-5. Test Pit is approximately 8 feet deep.

Test pit terminated at 6 feet below grade.
No groundwater seepage encountered during excavation.